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EFFLUENT DIFFUSION IN LARGE BODIES OF FRESH WATER

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OF

FRESH WATER

By:

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SUMMARY

This report consists of two Parts. Part 1 is a presentation of the implications and factors involved in fresh water disposal of waste effluents, and is the result of a review of current literature. Because of the sparsity of literature dealing with fresh water conditions, an attempt has been made to modify and adapt information dealing with ocean outfalls to comply with fresh water conditions. The main effective differences between the two respective bodies of water being density and circulation.

Part 2 of this report is the result of physical measurements of mixing and diffusion characteristics of the Toronto, Lakeview WPCP outfall site. This study was designed to test the application of the theories of fresh water diffusion as developed in Part 1 of this report as well as to determine the influence of the Lakeview WPCP waste on the water quality at the New Toronto and the Toronto Township water intakes.

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PART I

FRESH WATER DISPOSAL OF WASTE

1.1.0 INTRODUCTION

It is the purpose of this Part to describe the implications involved in fresh water disposal of partially treated wastes, while outlining the limnetic factors and measurements required for the effective design and location of any proposed outfall.

The information presented herein is the result of a literature search carried out as an initial stage of a research project on the diffusion of sewage effluents in large bodies of fresh water.

At present, the engineer is grossly handicapped in the rational analysis of many aspects of the outfall disposal problem because of a general lack of basic data. Consequently, much of the engineering of outfall installations has been based on judgement, limited past experience or on the results of crude surveys.

To make effective steps possible for improving the dilution method of disposal, a complete understanding of all the factors governing this method of disposal must be realized.

1.2.0 GENERAL

Domestic sewage and industrial wastes may contain any number and variety of polluting substances. These substances may have the capacity for causing, in the receiving waters, deposition of solids with resulting decomposition and release of offensive odours, depletion of dissolved oxygen necessary for aquatic life, production of foul and septic conditions and objectionable scum on surfaces and shorelines. In addition, these substances may reach concentrations toxic to fish and wildlife and may create a menace to human health.

The underlying principle of outfall disposal of waste is, therefore, the economic disposal of waste without producing any detrimental effects on the receiving water that will significantly impair its quality. In fresh water disposal this is accomplished by the effective intermingling of the waste effluent with the water body to achieve the following purposes: (1) the effective oxidation of the fine suspensoids and the dissolved organic compounds, (2) the reduction of bacterial densities, (3) the prevention of odour nuisance, (4) the removal of particles by sedimentation with natural detrital bottom deposits, and (5) the dilution of toxic substances to tolerable concentrations. Thus, there is a limit to the amount of waste that a given body of water can effectively assimilate.

Factors affecting the design of an outfall are numerous and complex, as many overlapping elements enter into the dispersion and dilution processes of the waste in the receiving water. Determining the dilutions necessary is often complex and difficult, especially where potentially toxic substances are being considered.

The design of the outfall begins with determining the most economical degree of treatment to be achieved by the plant. Preliminary surveys may be valuable in this respect. The beneficial uses and location of swimming areas and water intakes of the receiving water must be determined by domestic

and industrial surveys of the area, and its present quality by sanitary surveys. Limnetic 'in situ' measurements of currents, topography, and temperature and stratification profiles must be made to determine obtainable dilutions. The most effective design and location of the outfall and diffuser may then be determined and criteria developed to determine whether or not the disposal will be adequate.

Good engineering, together with a resourceful limnetic study of the proposed disposal site, will ensure that receiving waters and beaches are kept free of pollution.

1.3.0 DESIGN CONSIDERATIONS

The various factors to be considered in the design and location of a proposed outfall system are presented in Table 1. As has been stated, the underlying principle of any outfall design is the economic disposal of the waste while maintaining the quality of the receiving water commensurate with its highest beneficial uses. All design considerations must be geared towards this end.

The beneficial uses to which the water body may be subjected must first be determined. This may be accomplished through industrial and domestic surveys of the area, positioning of water intake locations in the near vicinity and determining the extent of bathing and other recreational areas. Once the beneficial uses have been determined, the water quality criteria necessary to determine the degree of dilution which must be accomplished by the outfall system may be developed.

The dilution obtainable for any given design and location of the outfall may be calculated after a thorough investigation of all the limnetic characteristics present has been made.

1.3.1 Beneficial Uses

The Boundary Waters Treaty of 1909 recognized the importance of preserving the quality of the Great Lakes boundary waters, as their value lies in the multiplicity of purposes for which they are presently utilized and their future uses. One provision of the treaty is as follows:

"The following order of precedence shall be observed among the various uses enumerated herein for these waters and no use shall be permitted which tends materially to conflict with or restrain any other use which is given preference over it in the following order of precedence:

1 - Uses for domestic and sanitary purposes,

2 - Uses for navigation, including the services of the canals for the purposes of navigation,

3 - Uses for power and irrigation purposes."

Other water uses such as for industry, for recreation, and for the support of fish and wildlife, have come into prominence in the passing years and must now also be recognized.

Although the above uses have been specifically applied to the boundary waters of Canada and the United States, they may well be applied to any large body of fresh water.

Indirectly linked with the beneficial uses of the receiving waters are aesthetic considerations which are now growing in importance as guide lines in this method of disposal. Besides the possible nuisance condition of odours and floating debris, the question here arises as to whether or not an individual is content to swim in a disinfected sewage-water mixture or to drink water that has originated in such an environment.

TABLE 1

Factors to be Considered in the Design of Limnetic Waste Disposal Systems

1 - Beneficial Uses

1) Domestic and Sanitary purposes	5) Industrial and Commercial Uses
2) Navigation	6) Fishery and Wildlife
3) Power and Irrigation	7) Waste Disposal
4) Recreation	8) Other

Table 1 - continued

II - Water Quality Criteria to Protect Beneficial Uses	
1) Public Health a-coliform b-other	3) Aesthetic and Nuisance a-floating materials b-odours c-colours d-other
2) Industrial and Commerical a-temperature b-grease and oil c-suspended and settleable solids d-plankton e-other	4) Fishery a-toxic substances b-oxygen depressants c-antagonistic substances d-other

III - Limnetic Characteristics of Outfall Site	
1) General water circulation system	4) Density structure
2) Current a-strength and direction b-effect of wind and waves	5) Wave Effects
3) Eddy diffusivity and dispersion characteristics	6) Topography
	7) Geology

IV - Waste Dispersion Considerations	
1) Initial mixing - diffusion a-jet mixing b-buoyancy-gravitational mixing c-density gradients-thermoclines d-diffuser orientation e-waste dilution-flow continuity f-port selection-size, spacing	2) Waste transport-dispersion a-current regiment b-eddy diffusion c-mixing depth d-rational dispersion eqns. conc. dilution only conc. including decay

1.3.2 Water Quality Criteria

It is a relatively simple matter to determine the beneficial uses of a receiving water. What is difficult, is the determination of the effects that the waste may have upon these beneficial uses. If the waste outfall is to be effective, criteria must be developed to determine these effects.

Anything which materially affects the limnetic resources of the water will in some manner affect the majority of its uses. While it is doubtful that any sizable discharge can be made to a receiving water without affecting the biological character of the area, it is the engineer's responsibility to ensure that the effect is minimized as much as possible. Adequate dilution must be achieved so that any gross change in the normal flora and fauna is avoided.

What must be done to avoid significant deleterious effects of the waste discharge on the aquatic resources of the area is a difficult and complex question. It is obvious that the discharge of toxic materials must be carefully controlled and that the concentration of oxygen depressants must be limited, but what other factors must be considered?

Of foremost importance in the engineering analysis of outfall disposal of treated waste is the protection of public health. The public health aspect in some way affects every body of water and the lives of all those who come in contact with it. Beneficial uses of the water which are directly related to public health include bathing, general recreational uses such as boating, fishing, etc., its' use as a working environment and greatest of all, its use as a domestic water supply. The complex nature of the public health aspect and the criteria involved will be discussed in detail later.

The criteria required to protect the industrial and commerical uses of water are, in turn largely dependent

upon the specific application of the water and must be determined in conjunction with surveys of the area. These criteria are far too numerous to be discussed in detail in this report but have been briefly outlined in Table 1.

The relative importance of aesthetic and nuisance criteria may be determined from sanitary surveys of the area, including the location of water intakes, bathing and recreational areas. Odour, colour and taste may be important criteria for domestic water supplies as additional costly equipment is required for their removal.

Other criteria may be purely observational, such as the visible evidence of floating solids and oil or grease slicks.

1.3.3 Public Health Aspect

In general, water criteria related to public health may be established for essentially three beneficial uses of fresh water. They may be categorized as waters for: 1) domestic purposes, 2) general recreational purposes, including fishing and the working environment, 3) bathing purposes.

Public health considerations usually imply two types of analysis. First a sanitary survey to establish the physical presence, or possible presence, of fresh sewage in the receiving water is required. The second analysis involves the familiar coliform determination as described in "Standard Methods". When waters that have been polluted are to be used for municipal or bathing purposes, the progress of bacterial self-purification as measured by the prevalence of the coliform group of organisms may be the principal determinant. The concentration of coliform organisms reflects: 1) the relative hazard of infection incurred by ingesting the water, and 2) the degree of purification to which the water must be subjected before it can be used with safety.

The principal effect of treatment on coliform bacteria is to reduce the numbers of bacteria discharged to the receiving water and to reduce, correspondingly, the downstream densities. The type and degree of treatment of domestic sewage will greatly influence the number of bacteria entering the receiving water. Table 11 presents the bacterial removal efficiencies of the various treatment processes. Chlorination should be included as a polishing process in any treatment where outfall disposal of the effluent is being considered.

Table 11

Bacterial Removal Efficiencies of Sewage Treatment Processes

Treatment Process	Bacteria % Removed Range	Bacteria % Removed Midpoint	Bacteria % Remaining
Plain Sedimentation	25-75	50	50
Chemical Precipitation	70-90	80	20
Trickling filters*	90-95	92.5	7.5
Activated sludge*	90-98	94	6.0
Chlorination			
raw or settled sewage	90-95	92.5	7.5
Biologically Treated	98-99	98.5	1.5

*Preceded and followed by plain sedimentation

Once the effluent reaches the receiving water there is usually a slight increase before the decrease of bacterial concentration begins. The increasing phase usually continues until a maximum density is reached in about 10 to 12 hours in the summer and up to 60 hours in the winter. At the point of maximum density the total number of coliforms may be 4 to 8 times the number discharged in the outfall. This increase has been attributed to the fact that the dilution of the waste with the relatively clean lake water temporarily upsets the predator-bacteria balance and permits the bacteria to increase through multiplying more rapidly than the predators.

The rate of bacterial decay is dependent upon many environmental factors. Those having the greatest significance appear to be: 1) temperature of the receiving water; 2) predator (protozoa) bâlance; 3) availability of food supply and 4) presence or absence of toxic substances. Bacterial removal is also affected through sedimentation.

Bacterial Die-off Rates: Considerable research has been carried out on bacterial die-off rates, by various investigators. The bulk of this work however, has been carried out on rivers where the rates would tend to be somewhat higher than in the lake environment.

Scarce, et al, (1964) performed studies to determine the sürvival patterns of the coliform and fecal streptococcus bacteria present in raw sewage and treatment plant discharges, following mixture of these discharges into Lake Michigan waters. Figures 1 and 2 are redrawn from their data. As may be seen from Figure 1, they found that zones of poor water quality may persist in the lake over periods of many days. Certain pathogenic organisms may persist even longer than the indicator organisms.

Kittrell and Furfari (1963), review the findings of earlier investigators and present their own findings on coliform bacteria concentrations. Table 111 presents a comparison of rates of coliform decreases in several streams. Examination of this table would indicate that 90% reduction is achieved in 2 to 4 days time. Because of the higher dilutions obtained in a lake, this die-off rate will be considerably reduced and a greater decay period will be experienced.

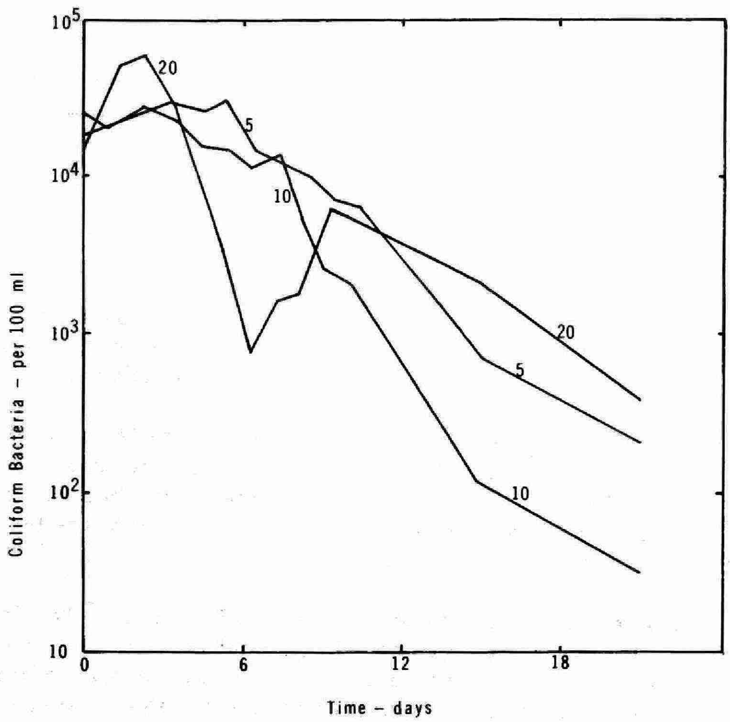


FIGURE 1 : Survival pattern of unchlorinated coliform bacteria introduced into Lake Michigan water and incubated at the indicated temp. -°C

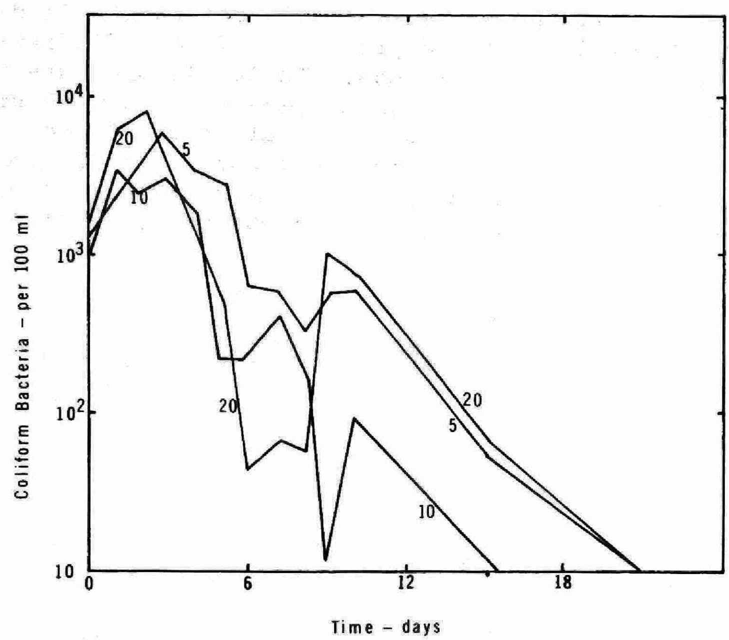


FIGURE 2 : Survival pattern of chlorinated coliform bacteria introduced into Lake Michigan water and incubated at the indicated temp. °C

Table 111

Comparison of Rates of Coliform Decrease in Several Streams

Source of data	% Remaining after Selected Intervals (days)			
	1	2	4	8
Hoskins (winter)	25-40	12-21	4.5-8.5	2.2
Missouri (winter)	50	30	13	-
Hoskins (summer)	14-26	4-12	0.6-2.2	.03-0.2
Tennessee River	25	7.4	0.95	0.07
Sacramento River	17	4.8		
Cumberland River	3.6	1.3		

The paucity of experimental data from fresh water lakes indicates the need for further research in this field. However, from available literature, it would appear that a value of t_{90} of 12 days would be a conservative figure under normal conditions. The die-off rate, k, may then be taken to be $1/t_{90} = 0.083$. Because of the many factors affecting die-off rate however, this figure should only be used in the absence of experimental data.

1.3.4 Effect on Lake Resources

From the viewpoint of the theoretical conservationist, it is improper to add only waste material to the waters of the land that might affect the natural balance of the environment, however small. Such a viewpoint is not at all realistic.

If wastes are discharged to lake waters under properly controlled conditions, much of the organic matter and trace nutrients present in the waste ultimately becomes available as food to the aquatic organisms. In certain environments where natural nutrients may be limited, properly controlled waste discharges may actually stimulate the biological activity of the area. Generally, however, this

increased productivity is detrimental to the lake and tends toward oxygen depressed areas or over-productive areas with high plankton concentrations which may appreciably affect water usage.

1.3.5 Possible Harmful Effects

Pollution of the water body may be measured physically, chemicallyand biologically. No single parameter tells the entire story. Depending upon the nature of the polluting substances and the uses that the receiving body of water is to serve, harmful effects may be related to such parameters as colour, turbidity, odour, nitrogen in its various forms, biochemical oxygen demand (BOD), dissolved oxygen and other gases, mineral substances of many kinds, bacteria and larger aquatic organisms.

Physical: Physical effects on the receiving water depend primarily upon the outfall location and the character of the sewage discharge.

TEMPERATURE changes due to waste discharges may be of some concern under certain conditions where a cooling water intake is located in close proximity to the outfall. Generally, however, local temperature changes are of little significance where large bodies of water are concerned, because of the considerable temperature changes that naturally occur.

DEPOSITION OF SOLIDS is generally insignificant where secondary treatment is employed. The rate of deposition is however, dependent upon the topography and must be considered under certain circumstances. Excessive deposition of solids on the lake bottom may restrict the normal benthic organisms.

TURBIDITY may develop in the proximity of the outfall due to high contents of suspended solids in the discharge. The turbidity may be so fine that a relatively stable suspension is formed. This condition may be lessened by either improving the treatment process or by improving the design of the diffuser to obtain greater initial mixing and dilution.

Turbid conditions may also develop due to the increased biological activity of the area. As indicated earlier, the waste may stimulate the growth of plankton in the receiving waters. The relative importance of this effect must be determined from the beneficial uses of the water.

The COLOUR, TASTE and ODOUR of the receiving water may be affected by the outfall, especially by those handling industrial wastes. Industrial wastes may contain dyes and other colouring substances which may be quite harmless but nevertheless undesirable from an aesthetic point of view.

Taste and odour in water is caused by volatile substances associated with organic matter, living organisms and gases such as hydrogen sulphide. The chlorination of water may produce tastes and odours of its own or intensify those of odour-producing agents. Tastes, odours and colour may also be reduced by improvements in treatment and outfall design.

Chemical: Normally, outfall disposal of waste has little effect on the chemical characteristics of the receiving water. Certain industrial wastes may contain some substances that will produce chemical changes in the receiving water; however, if dilution is great enough these effects should be negligible.

Of considerable concern under some circumstances is the depletion of the dissolved oxygen in the receiving water. Again, however, the high oxygen demand is generally restricted to the near vicinity of the outfall since waste-water dilutions much greater than those required by oxygen balance are usually obtained. This effect is largely dependent upon the oxygen demand of the waste and upon the condition of the receiving water. If, for some reason, the dissolved oxygen content of the receiving water is low initially, the effect of a high BOD waste may augment this condition. If secondary treatment has been provided and the diffuser is adequately designed, no difficulties in dissolved oxygen should be felt.

Biological: The only adverse biological effects of significance are toxic effects involving some members of the normal fauna and flora. Domestic sewage as such has relatively few toxic characteristics that might adversely affect the environment. However, industrial wastes may contain a variety of toxic materials such as chromium, nickel, copper or cyanides. The toxic effects of the waste may be estimated from analyses for presumed toxic constituents and expected or attained dilutions.

1.4.0 MECHANICS OF DILUTION

In order to understand more fully the various field measurements and determinations required in designing and locating a waste effluent outfall, a thorough knowledge of the mechanics of dilution and dispersion is required. Besides improving the physical appearance of the waste effluent it is also necessary to reduce the bacterial concentrations in the effluent receiving waters to safe and acceptable limits. Moreover, concentrations of deleterious or toxic substances must be maintained at sufficiently low levels to prevent damage to the flora and fauna of the receiving water.

In order to economically meet the above requirements for effluent disposal with lake outfalls, it is necessary to consider the most effective methods of mixing and dispersion of the effluent in the receiving water.

There are essentially two major fundamental aspects of the dispersion problem. The first is concerned with the mixing and dilution of the waste effluent in the immediate proximity of the discharge point. The second is associated with the ultimate disposition of the effluent-fresh water mass. Of principal concern in the latter is the direction and persistence of movement of the mixture as well as the mechanics of the attendant dispersion process.

When the effluent is discharged to the water, it is immediately subjected to a buoyant force proportional to the density differences. This force deflects the jet towards the surface and the relative motion between the jet and the water body develops shear stresses. Turbulence is generated and mixing takes place throughout the entire column. As mixing progresses, density differences are decreased and the vertical driving force is reduced. Depending upon depth, stratification and initial density differences, the waste field may or may not reach the surface. Superimposed upon this vertical rise is mass movement due to any water motion.

1.4.1 Initial Mixing and Dilution

The design of the diffuser section of the outfall plays a major role in achieving the full benefits of the initial mixing process. Rawn and Palmer's studies (1930), as well as the arguments presented by the discussers, confirm the obvious advantage of multiple-port diffuser sections to achieve higher initial dilutions of the effluent in the receiving water.

Diffuser sections with vertical, inclined or horizontal outlets or ports are in use at present. However, because of the increased dilution obtainable from the horizontal port, horizontal ports only will be considered here.

The fluid mechanics of a single outlet discharging into a body of water with a density different than that of the jet is complex, and becomes even more complex when variations of density occur in the water body itself. Since neither the precise forces nor the characteristics of the flow pattern are known, the forces on the trajectory may only be approximated.

A comprehensive literature review as well as an authoritative discussion of past research on initial mixing and dilution has been presented in Pearson's study (1956). Many formulations for calculating initial mixing and dilution have been presented.

With fresh water as the receiving body of water, it would appear that the original Rawn and Palmer studies would be most adaptable for determining waste dilution. Initial jet mixing and resultant dilution are accomplished through the turbulence created by the jet acting upon the receiving water and are therefore a function of viscosity, depth of discharge - Yo, diameter of jet - D, jet flow - Q, and the apparent acceleration due to gravity, g or g'. Rawn and Palmer arranged these variables into three dimensionless parameters to determine dilution, such that:

$$So = f\left| \frac{Yo}{D} , R,F \right|$$

A re-evaluation of the original data was made by Rawn, Bowerman and Brooks (1960), with respect to Reynolds number and there was no indication that So depends on Reynolds number in the range of R from 5,000 to 40,000. As long as the flow in the jet is well within the turbulent range, the data indicated that Reynolds number had no significance. This confirms observations by Falsom and Ferguson (1949), Albertson (1948) and Schlichting (1949). Thus the dilution So is a function:

$$So = f\left|\frac{Yo}{D},\ F\right|$$

$$\text{where } F = \frac{Q}{\frac{\pi}{4}D^2\sqrt{\frac{\Delta\rho}{\rho}gD}}$$

The relationship can best be shown graphically as in Figure 3 which has been drawn from Abraham (1962) and covers a wide range of values based on a combination of theoretical and experimental results.

Initial jet mixing then is affected by lake conditions, only in respect to density differences between waste effluent and the receiving water and in density stratification of the receiving body. It is largely dependent, however, upon the outfall diffuser design.

Initial dilution is also dependent upon the availability of fresh water in the vicinity of the diffusers. This aspect may be estimated from the ratio of the total flow past the diffuser to the discharge volume, i.e.:

$$So = \frac{Ubh}{q}$$

where U is current velocity, b is the effective diffuser length and h is the average thickness of waste field and may be taken as 1/4 of the depth if no thermocline is present.

Where there are many ports discharging from a diffuser, there will be some interference between the flow patterns established by the individual jets. This interference will tend to reduce the dilution somewhat below that which could be obtained from a single port. A reduction in the order of 30% has been suggested by Whitt (1965), depending upon the spacing, port size, etc.

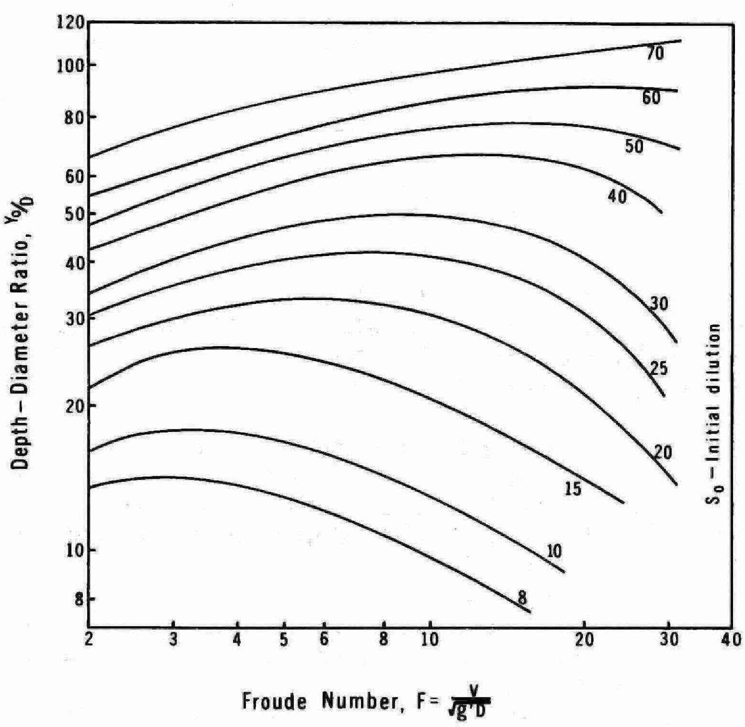


FIGURE 3 : Initial dilution for submerged horizontal jet discharge.

1.4.2 Turbulent Diffusion of Waste Field

Because the initial mixing and dilution achieved by the diffuser section generally does not dilute the waste effluent to a harmless level at the outlet it is necessary that subsequent dilution processes be considered.

As the effluent field moves along in the direction of the current, mixing takes place along the edges of the field, and the initial step function for the concentration c (x y) is gradually spread out as in Figure 4. The Fickion diffusion law, with a variable diffusion coefficient, may be said to apply to this process. Thus, the formulation of the equation by Brooks (1965):

$$c_{max}\ (x) = c_o \operatorname{erf} \sqrt{\frac{3/2}{(1 + \frac{2}{3}\beta \frac{x}{b})^3 - 1}}$$

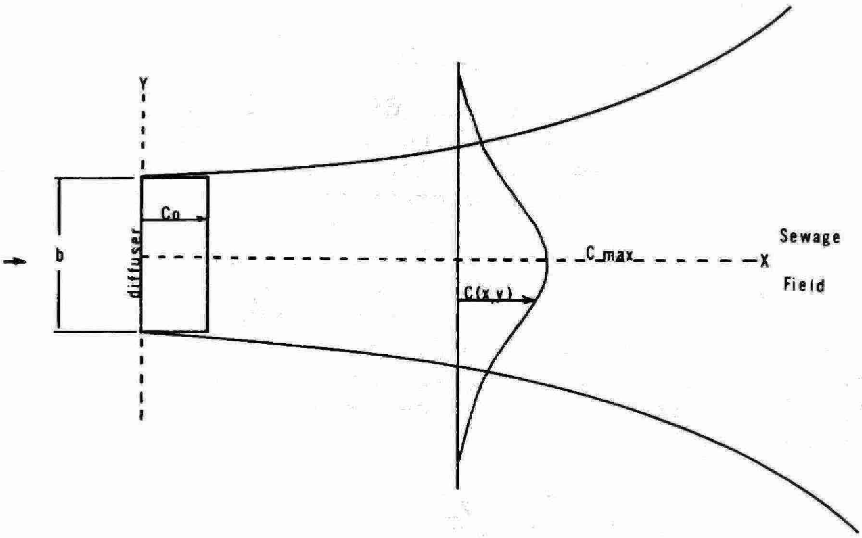
which gives the concentration along the centerline of the field at a distance (x) from the source. β is a dimensionless unit and related to the initial diffusion coefficient Eo as

$$\beta = \frac{12Eo}{U\ b}$$

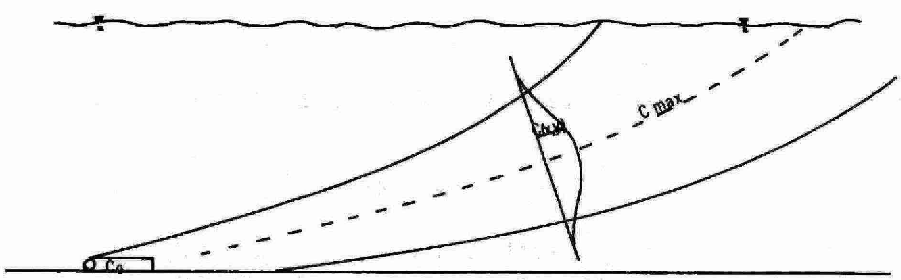
The solution of this relationship is given by Figure 5.

The difficult step in estimating this process of dilution is the determination of the diffusion coefficient.

Considerable research has been carried out in determining the coefficient of eddy diffusion by various investigators, i.e. Schmidt (1925), Ruttner (1963), Munk, et al., (1943), Mortiner (1941-42) and others. Schmidt has determined the coefficient of eddy diffusion for four depths of Lunzer Untersee, a small lake in Switzerland, at various periods of the year.



PLAN VIEW



SIDE VIEW

FIGURE 4 : Schematic representation of possible diffusion pattern from a horizontal diffuser port

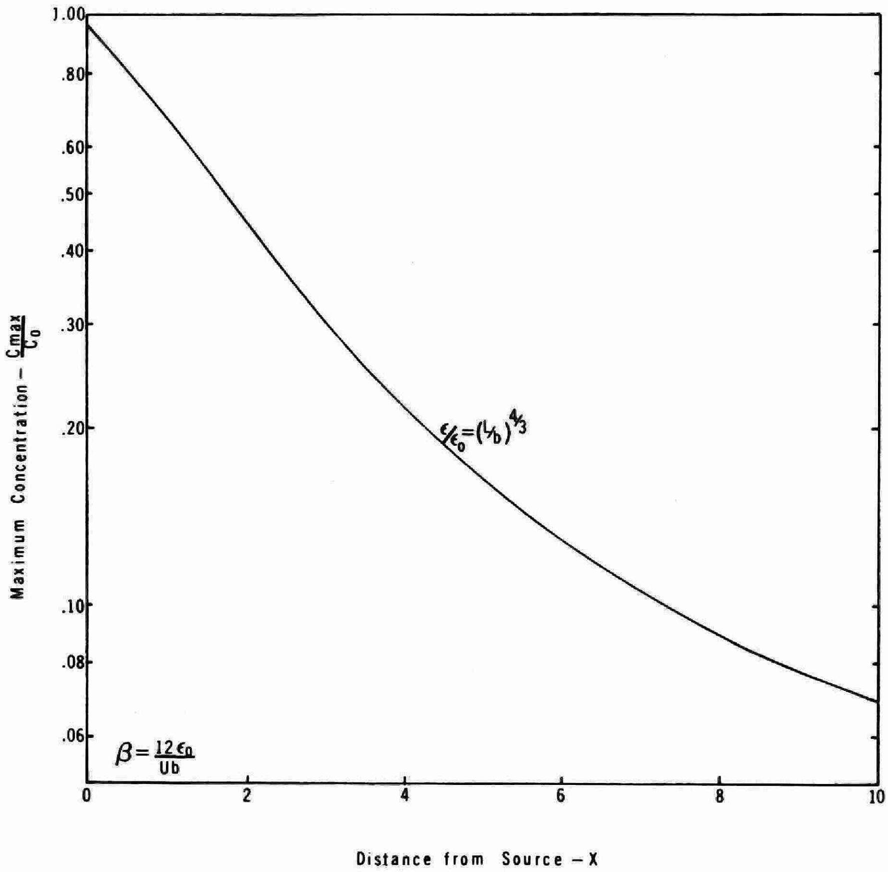


FIGURE 5: Dilution along the centerline of a sewage field in a lake current according to Richardson's 4/3 law.

Depths (M)	9/4 to 15/4	to 8/5	to 20/6	to 30/7
5	2.02 cm^2/sec	0.73	0.57	0.18
10	2.41	0.55	0.38	0.06
15	2.24	0.32	0.21	0.04
20		0.20	0.04	0.01

Consideration of these values is useful in several respects. First, it can be seen that the mean coefficient of eddy diffusion decreases with the time of year and with the depth of the water. Only in the first interval, which coincides with the spring turnover, is the coefficient uniformly high at all depths investigated. After the establishment of thermal stratification, the values show a general decrease and also a great difference with depth which is especially marked when summer stagnation has set in completely. It can be seen how mixing is greatly restricted by a fully developed metalimnion; but in spite of this there is still a slight degree of turbulence in the hypolimnion.

Csanady (1964) determined the horizontal coefficient of eddy diffusion in the upper 1 to 2 meters. He found the coefficient to range from 250 to 2,000 cm^2/sec on relatively calm days with strong solar heating and slow evaporation, indicating a minimum of surface turbulence.

Although not directly a result of dilution, an additional consideration must be made if the pollution factor under question is subject to die-off. To obtain this concentration reduction the value of $\frac{Cmax}{co}$ from Figure 5 is simply multiplied by the factor $c^{-kx/V}$ where k is the die-off rate and V is the current velocity.

1.4.3 Effect of Stratification

If the receiving water is stratified due to variations in temperature, the flow pattern may be considerably altered.

This stratification may limit the rise of the effluent jet and maintain an effluent field at some depth greater than would occur in the absence of stratification. The graphical relations in Figures 3 and 6 can be used to determine the approximate value of dilution at the level at which the waste in the cold water rises to meet the warm water interface. If S is the specific gravity of waste and Sw and Sc the specific gravities of the warm and cold layers of lake water respectively, then the mixture will stay submerged below the warm surface layer if

$$\frac{(S-1)\ Sc + S}{S} > Sw$$

In addition, the density of the resulting mixture should be such as to overcome the residual inertia of the rising column.

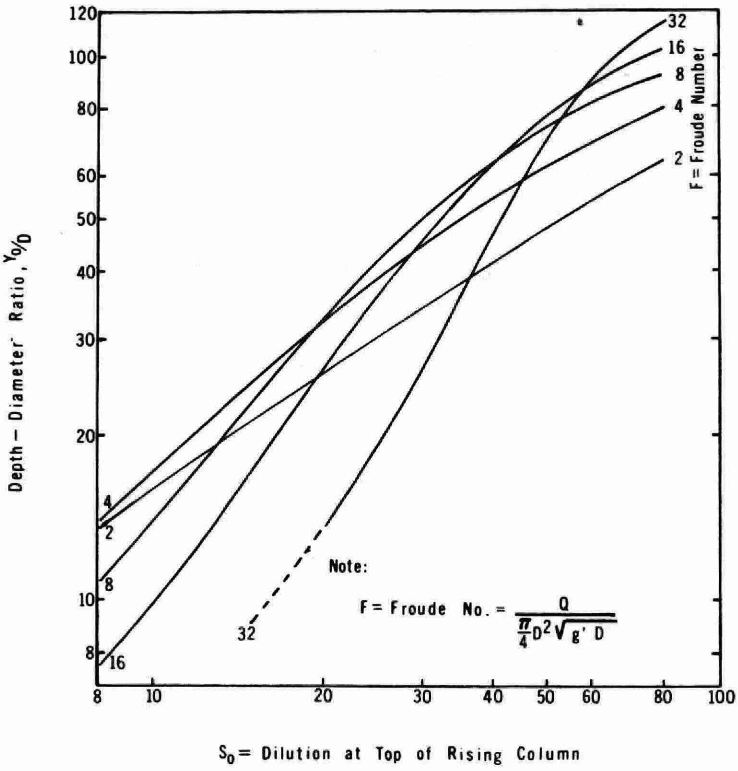


FIGURE 6 : Dilution as a function of y_o/D for constant F for horizontal discharge (based on Rawn-Palmer data)

1.5.0 LIMNETIC CHARACTERISTICS AND MEASUREMENTS OF OUTFALL SITE

The limnetic data required for properly engineering an outfall system differ considerably, depending upon the type of waste to be treated and the degree of treatment proposed. In some cases it may be advisable to make preliminary lake surveys to establish the degree of treatment necessary to ensure adequate disposal. The maximum degree of treatment is desirable in view of the extended use of this method of effluent disposal.

Physical, chemical and biological determinations are required. Biological and chemical data are necessary to determine the present condition of the receiving water. The importance of such data lies in determining the dilutions required by the outfall to render it effective. Physical data is required to determine the dilutions possible through a properly designed outfall.

Chemical measurements include determinations of dissolved oxygen concentrations and concentrations of any chemical pollutants that may already be present in the receiving water. Biological measurements may be limited to determining the productivity of the water in relation to plankton and larger species of plant and animal life.

Physical measurements are much more involved than those of a chemical or biological nature and are of much greater importance in the engineering design of the outfall. Fundamentally, there are three types of physical measurements required. They are topography, temperature and stratification profile and current measurements, each being of equal importance but varying in complexity.

1.5.1 Topography

Bottom topography data and sediment analyses are important in choosing the location of the proposed outfall system. In areas where currents may be most favourable for dilution and disperson the bottom conditions could be unfavourable, requiring costly excavation or elaborate lateral support for the outfall pipe.

Shoreline topography is also important in predicting mass transport of water. If the shoreline is smooth, currents will tend to flow parallel to the shore in one direction or the other, resulting in large mass movements of water. If the shoreline is highly irregular intricate current patterns may result with little mass movement in certain areas.

1.5.2 Temperature and Stratification

Measurements of temperature patterns and differentials give the average daily vertical location and strength of the thermocline if such stratification exists. It is important to know these relationships for all seasons and under all weather conditions. During a storm, with accompanying high winds, a complete absence of temperature differential is usually found because of the thorough mixing. During these periods of complete mixing, dilution and dispersion may be so great that no definite sewage field can exist. Prompt dilutions in the order of 1,000 are possible with adequate diffuser design and depth. In contrast during periods of extended calm and high or low temperature, complete stratification of the lake depth may result. Under these conditions, minimum dilutions occur and the diffuser design and location become all important.

Considerable work has been done in determining the thermal structure of the Great Lakes and the results obtained may well be applied, with modifications, to any large body of fresh water. Rodgers and Anderson (1963) carried out extensive studies on Lake Ontario. They found that during the winter or late cooling stage, Lake Ontario is thermally the most homogeneous and until April, temperatures are uniform at about 4°C or increase very slightly with depth to a maximum of 4°C. By the middle of June, a thermocline has been established at a depth of 20 to 30 feet with a surface temperature of about 50°F. By early fall surface temperatures reach a maximum of about 70°F but the thermocline has remained at approximately the same depth. During late fall the thermocline deepens quite rapidly until it finally disappears in the winter season.

Coincidental with this horizontal stratification is a vertical stratification process which follows a similar temporal pattern, see Figures 7 and 8.

In winter when all lake waters are less than the temperature of maximum density (4°C), the vertical circulation is undoubtedly aided by indifferent stability in the water and by thermal convection. However, in the spring and fall when there are areas of water both above and below 4°C, vertical circulation induced at the boundary between these two water masses by production of denser water may exert control on the horizontal movement of water. This region of mixing, which has a temperature close to 4°C, has been called the 'thermal bar' by Tikhomirov (1963). He suggests that the thermal bar is a feature common to all dimictic lakes of the temperate zone. Data demonstate or imply its existence in all the Great Lakes and it may possibly be found in other inland lakes.

The thermal bar as defined can exist only when masses of water both above and below the temperature of maximum density are present. Therefore, it does not exist in mid-winter when all the lake is at a temperature less than 4°C nor can it exist when the lake consists of water wholly at or above the temperature of maximum density.

Thermal bars occur both in the spring and in the fall.

The progress of this stratification process is highly complex but may be outlined quite simply. As spring heating progresses, nearshore waters heat to 4°C before the deeper central regions. Warm runoff waters probably contribute to the nearshore warm water in the early stages. A thermal bar forms near the shore as a boundary between the midlake waters, less than 4°C, and the warm inshore waters. During the following weeks as the heating progresses, the thermal bar moves towards the middle of the lake. As the bar moves off-shore the central region maintains a near uniform vertical homogeneity and in the region around the shore a thermocline develops.

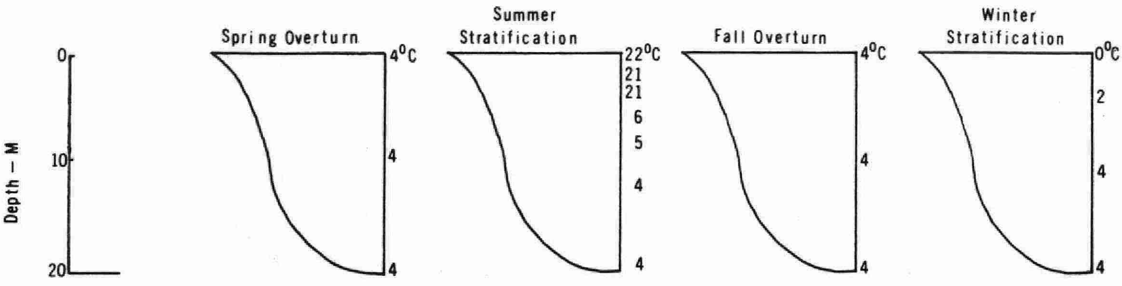


FIGURE 7 : Horizontal stratification in a temperate lake

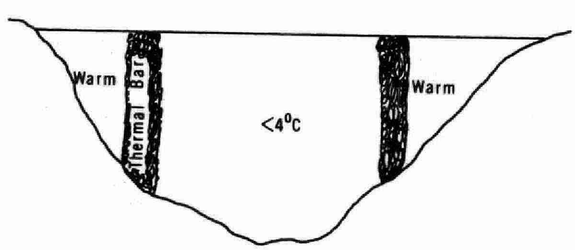


FIGURE 8 : Spring movement of a thermal bar

Surface water convergence and sinking is implied at the bar where contraction due to mixing of waters less than and greater than the temperature of greatest density takes place. There is vertical isolation of horizontal water movements in the central and nearshore regions. With sinking all around the outside edge of the central core, upwelling in the middle is induced.

Under fall conditions, the circulation pattern of the cold and warm water masses would be reversed.

The thermal bar when close to shore in spring appears to impound runoff and probably limits the dissipation of "pollution" injected into the zone between the shore and the thermal bar. Sampling close to shore in spring and early summer will not be representative of the entire lake contents.

Another form of stratification may develop in the upper few meters of water under conditions of the absence of both wind stress and surface cooling. Surface temperatures rise under these conditions and become much less dense than lower regions. Such stratification is helpful in maintaining a submerged sewage field when turbulence is at a minimum.

1.5.3 Currents

The total collection of forces, both internal and external, which determines the behavior of ocean, sea or lake currents makes it almost impossible to present a clear-cut theory on current processes. Each relevant factor must be treated individually in an approximate fashion and each new location must be investigated separately. Generalizations, however, may be developed for specified conditions.

Currents may be said to be caused by wind action, gravity and gradient forces, and by hydraulic effects. These forces are modified by the Coreolis' Forces, bottom topography and shoreline contours. The relative significance of each varies with the time of year, depth of water, surface area of water body, proximity to shore and to locations of inflow and outflow and intensity of stratification.

In actual observations one may never observe a pure hydraulic current pattern, a pure wind drift current pattern or a pure gravity current pattern, but rather a superimposition of all three. Hydraulic currents are present wherever there is water flowing into or out of a lake, however localized their effects may be. A pure wind drift current, after some time, gives rise to an opposing gravity current due to 'piling up' of the water. If the wind continues, the drift current is maintained near the surface while near the bottom there exists a return gravity current which will continue to grow until it exactly counterbalances the piling up effect of the drift current. However, as will be seen later, gravity currents may also occur in the absence of wind drift. With a thermocline imposed, far more complicated combinations become possible.

Hydraulic Currents: Hydraulic currents are caused by the flow of water into and out of a basin, and could perhaps be classified as a type of gravity current. The relative importance of this current depends upon the size of basin and quantity of flow. The smaller the lake and the greater the flow, the more profound is the influence. In the Great Lakes, it would appear that the effects of flow are mostly localized, occuring at the mouths of rivers and in the vicinity of outlets. This fact has been substantiated by observations of Buretta (1964), Ayers, et al., (1958), Maguire (1961) and others.

In smaller lakes with high flows the hydraulic effects may be quite significant.

The path of inflowing current is fixed by the primary direction of the water course, the formation of the basin and by the direction of rotation of the earth. Its depth is dependent upon the density relationships of the water masses, being generally within the metalimnion if such is present, but changing with the season. Outflow from a lake generally involves only the surface layers.

Wind Drift and Gravity Currents: If we neglect, for a moment, the hydraulic effects, two ideal cases arise: 1) the

"pure wind driven current" (drift current) and 2) the "pure gravity current". The former occurs when only the wind stress, the Coreolis' forces and friction are acting. In the latter case the wind stress is replaced by gradient forces. Buretta (1964) has formulated equations for both pure drift currents and pure gravity currents and the Ekman spirals for both cases are presented.

When considering the wind drift effect, both the frictional and Coreolis' forces are taken into account, the end result being that the wind drift at the surface is directed 45 degrees to the right of the wind direction. As depth increases, the angle between wind and current directions decreases and also, of course, the magnitude of the velocity decreases. The pure wind drift current can develop only in deep water in regions where the wind blows in the same direction and with the same velocity over a wide area and in waters of a uniform density. Thus, wind drift as such, is of little consequence in respect to waste diffusion in relatively shallow waters and in areas close to shore.

The existence of drift currents gives rise to gravity return currents in that mass transport results in a piling up of water at the shoreline. The lake surface then becomes inclined, inducing its own characteristic water movement. Thus, near shorelines and in regions of wind shift it is the gravity current, the secondary effect of the wind, which becomes important in outfall design. As explained by Maguire (1961), application of the Coreolis' effect to currents related to density distribution leads to the rule that currents in the Northern Hemisphere flow in such a direction that the water of low density is on the right and the water of high density is on the left-hand side of the current. Near the shoreline, then the secondary and significant effect of a wind blowing parallel to the shore over a prolonged period of time is to develop a current which is parallel to the shore in the direction of the wind. For the same reasons, winds blowing on or offshore maintain currents which flow parallel to the shore.

The above theories were tested in Lake Huron (Ayers, et al., (1956) and in Lake Michigan (Ayers, et al., 1958). Evidence of the primary relationship between wind and current was more obvious in the Lake Huron study where winds prior to the cruises had greater tendencies to be unidirectional. Evidence of the secondary relationship was noted in the Chicago, Milwaukee and Grand Rapids areas during the Lake Michigan survey.

Currents in the upper layers of the water may also be caused by surface cooling, or evaporation under a hot sun. The degree of mixing which takes place is determined by the intensity and scale of turbulence: thermal and mechanical. The former is initiated by surface cooling, while the latter, as has already been dwelt upon, is either due to wind stress or bottom friction. Evaporation of the surface layer causes its cooling and its resulting increase in density, causing it to drop and to be replaced by warmer water. Circulation is set up causing high turbulence in the upper layers and greatly adding to diffusion of a waste if it should rise to the surface.

In the absence of both wind stress and surface cooling little mixing occurs and a stable stratification develops under solar heating. This further reduces the turbulence and gives rise to very complex current patterns, since each layer moves more or less independently of the others. Rodgers (1964) found, at times, a 90 degree difference in current direction between depths of 0 and ½ meters.

1.6.0 OUTFALL DESIGN

Sewage effluent discharged at an appreciable depth into a lake is immediately surrounded by the somewhat colder and denser lake water. This causes a vertical rise of the less dense sewage until adequate dilution has occurred to render the mixture of a common density and a submerged waste field is usually established. If, however, the diffuser has been inadequately designed, or depth has been insufficient, this submerged equilibrium is not reached and the effluent will rise to the surface. This is undesirable from an aesthetic and health point of view.

In a given outfall, improvement of dispersal of waste effluent is accomplished by the use of an outlet diffuser at the end of the outfall. Diffusers are necessary to obtain adequate dilution in fresh water disposal.

The data used for the outfall design should be adequate to allow the evaluation of average lake conditions for all seasons of the year. The following topics in design must be considered:

1) depth of discharge and length of outfall,
2) size and spacing of diffuser ports,
3) shape and orientation of diffuser.

Depth of discharge and length of diffuser: The depth of discharge of outfall is regulated by economic considerations associated with the proposed outfall. In many cases, lengths up to a mile or more would be required to reach a depth of 40 - 50 ft because of the gently sloping floor. Unit construction costs are higher in longer lengths because larger diameter pipes are required to overcome additional friction losses unless higher pumping heads are justified.

Up to a point, long diffusers in shallow water will produce dispersion efficiencies equal to those of short diffusers in deep water. Construction costs for many alternatives can be equated for a given required discharge capacity. Diffuser length is a function of diffuser depth, but as depth is decreased, the average density of water available for mixing also decreases, thus increasing the required dilution to maintain a submerged field. Therefore, a small increase in diffuser depth effects a larger decrease in diffuser length. The cost of construction of a longer line to increase depth is greater than the cost of additional diffuser construction, thus, these factors must also be balanced.

Size and Spacing of Diffuser Ports: The size of the diffuser ports greatly influences the resulting dispersion of the waste. Small diffuser ports spaced closely together will produce more initial dilution than larger ports spaced further apart. The most effective and yet simple type of diffuser is one which distributes the outflow through many ports over a large area with minimum head loss and interference between rising column.

The division of outflow between the various ports should be fairly uniform to achieve maximum dilutions. If the diffuser is laid on a sloping lake bottom, it will be impossible to achieve uniform distribution between ports for all rates of flow. In such cases, it is advisable to make the distribution fairly uniform at low or medium flow, and let the deeper ports discharge more than average during high rates of flow. This will prevent clogging of the deeper ports of the diffuser.

All ports should flow full in order to prevent the intrusion of receiving water into the pipe. Such intrusion will materially reduce the efficiency of the diffuser. According to Rouse (1946), the Froude Number should be greater than 0.59 in order for an orifice to flow full. A reasonable figure of $F>1$ may be applied.

The spacing between ports is rather inflexible, in as much as practical considerations dictate that the spacing be equivalent to the length of a pipe section or a simple multiple thereof.

Shape and Orientation of Diffuser: The shape and orientation of the diffuser with respect to the shoreline will depend upon the lake bottom profile and the prevailing wind and resulting current direction. Location of water intakes and bathing areas should also be considered as has already been suggested.

For further design data reference should be made to the California State Water Pollution Control Board publication No. 14.

1.7.0 CONCLUSIONS

With a good diffuser design, the amount of effluent that can be discharged into a given area depends primarily upon current velocity, density differential and quality of receiving water and the character of the waste. A thorough investigation of these conditions will effect sizable savings in the construction of large outfalls. Proper evaluation of these factors for all seasons prior to commencement of design is therefore a necessity.

Costs of limnetic survey work are minor compared to the savings in outfall costs that can be obtained by knowledgeable engineering design. Costs of effluent polishing or tertiary treatment facilities should be balanced against savings in construction costs for more elaborate outfall systems.

The most efficient diffuser would have horizontal ports with a high initial jet velocity and a large length-to-diameter ratio. However, higher jet velocities require higher discharge heads. The length of diffuser could be increased and the discharge head decreased, accordingly, to give equal efficiencies, within limits. Amortized expense of additional diffuser length could then be balanced with savings in pumping head costs over the life of the outfall.

It is important to keep the sewage field below the thermocline in summer, because currents above the thermocline could carry effluent swiftly into recreational waters. These currents, when directed towards the shoreline, usually deflect and parallel it at some distance offshore. Currents above the thermocline could thus cause problems in other areas depending upon local geographies.

Field work is presently being carried out on the Lake Ontario shoreline to improve the equipment and techniques for gathering limnologic data and to obtain a better understanding of the true relationships between limnetic factors and the design of outfalls.

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PART II

DIFFUSION OF LAKEVIEW WPCP EFFLUENT IN LAKE ONTARIO

2.1.0 INTRODUCTION

A study to determine the mixing and diffusion characteristics of the Lakeview WPCP outfall site has been conducted by the Applied Sciences Branch of the Division of Research, Ontario Water Resources Commission. The purpose of the study was to test the application of the theories of fresh water diffusion as developed in Part I of this report and to determine the influence of this waste on the water quality at the New Toronto and Toronto Township water intakes.

To this end, drogue releases were made during July and August, 1966 under various wind and water conditions, to determine mass movement of water in the outlet vicinity. The dispersion relationships were observed by means of both drogue and dye releases.

As a result of these studies, it was found that water movement in this particular location, in relation to both speed and direction is largely dependent upon wind conditions. Under calm conditions there appeared to be a mass movement of water in the westerly direction of no greater than 0.1 knot and decreasing in speed with depth. This underlying mass movement of water was by no means consistent as at times it changed directions by as much as 90^{o} for extended periods of time.

It was concluded that, depending upon wind conditions varying concentrations of Lakeview WPCP effluent could conceivably reach either the water intakes of New Toronto or of Toronto Township· however, dilutions of the effluent would be in the order of 99%. No appreciable harmful effects of the Lakeview WPCP effluent should be experienced at either intake location, even with the increase of flow (from 5 to 12 MGD) through the plant, provided that the treatment achieved by the plant remains equal to that at present.

2.2.0 THEORETICAL CONSIDERATIONS

One of the principle objectives of waste effluent disposal in any body of water is to dilute the waste by allowing it to mix with the surrounding water. This mixing and dispersion in general involves two aspects: the initial mixing in the immediate proximity of the discharge and the disposition of this initially mixed wastewater mass by such influences as lake currents and waves.

The first part of the problem depends on the type, design and location of outfall and on the characteristics of turbulence at the outlet. The second aspect depends on the natural turbulence in the lake, whether the effect of wind action, gravity and gradient forces or hydraulic effects. The relative significance of each varies with the time of year, depth of water, surface area of water body, proximity to shore, locations of inflow and outflow, and with intensity of stratification.

Under the action of most of these forces, dispersion of waste occurs in both the horizontal and vertical directions. While it is recognized that the mixing process is truly a three dimensional process, many investigators have neglected vertical dispersion as being small in relation to the horizontal dispersion. This assumption is reasonable in lakes with wind driven currents and where stability considerations modify the turbulent exchange processes in the vertical direction.

In the case of the nearshore area of Lake Ontario, however, there is normally very little stratification in the first 15 to 20 feet of water. Waste effluent, released at a depth of less than 20 feet, invariably being warmer than the lake water, will tend to rise until density differences are overcome by dilution. Thus some vertical dispersion is effected and a probable assumption of the vertical diffusion may be taken as being 2% of the horizontal diffusion, where distances of over one mile are involved.

2.3.0 STUDY METHOD

A complete determination of the diffusion processes of the Lakeview WPCP outfall site would require extensive field observations involving much equipment and personnel time. Since the processes themselves are so varied and irregular an estimate of the dilutions under weather conditions most adverse to dilution would be considered adequate. Formulations advanced through model experiments, empirical curve fitting and actual field installations have been developed by various investigators. These formulations are adequate to achieve first approximations of dilutions in the absence of field data.

In this study, field determinations have been limited to the determination of mass movement and relative velocities of water in the vicinity of the area of concern.

Preliminary dye and drogue releases were made to determine the forces involved in water movement in the Lakeview vicinity. Rhodamine B was chosen as the tracer dye because of its low natural background, low rate of adsorption and relative inexpensiveness. The drogues were constructed of two 2' x 2½' sheets of aluminum each, slotted and fitted together in the form of a cross and suspended from a small piece of styrofoam. These drogues could be suspended at any desired depth.

Once the general mass movement of water was established drogue releases were again made with two tracking stations being established on shore. The drogues were then located at regular intervals of time depending upon the speed and direction of drogue movement. Since the distance between the outfall and the Toronto Township water intake is less than 1.5 miles, the maximum drogue tracking time used was 9 hours.

2.4.0 FIELD OBSERVATIONS

Preliminary drogue releases were made at the Lakeview WPCP outfall on July 13 and 15, 1966 in an effort to determine the characteristic movement of the water mass in that area. Wind direction on these days was from the south and west, respectively, and since the drogues moved in the approximate direction of the wind, it was concluded that currents in this locality were largely due to wind stress.

On July 15, dye was released with the drogues. The dye remained at the water surface and moved considerably faster but in the same direction as the 5 foot drogues.

Drogues were released and traced at depths of 5 ft, 9 ft and 12 ft on July 19, 20 and 22. Wind direction on these days was from the NE at 15 to 20 mph, NNE at 15 to 20 mph and SSW at 5 to 8 mph respectively. (See Appendix A, Figures 11, 12, 13 and 14 for plots of drogue movement.)

Again it was observed that the drogues moved in the approximate direction of the wind but tended to swing back and forth laterally and did not follow a straight line. Figure 9 presents a plot of depth vs current speed as a percentage of wind speed.

Drogue and dye releases were again made on August 17 in order to obtain approximate dispersion relationships. The wind at this time was from the NW at 8 to 10 mph. Plots of the drogues and dye are given in Appendix A, Figure 15.

During the tracking of the drogues, it was noticed that they seldom moved in a straight line for any appreciable distance but tended to swing back and forth laterally. This phenomenon has been attributed to the internal movement of the lake proper. Thus, even under seemingly steady conditions of wind speed and direction, fluctuations in both current speed and direction are likely to occur. The trend of water movement, however, is in the general direction of the wind.

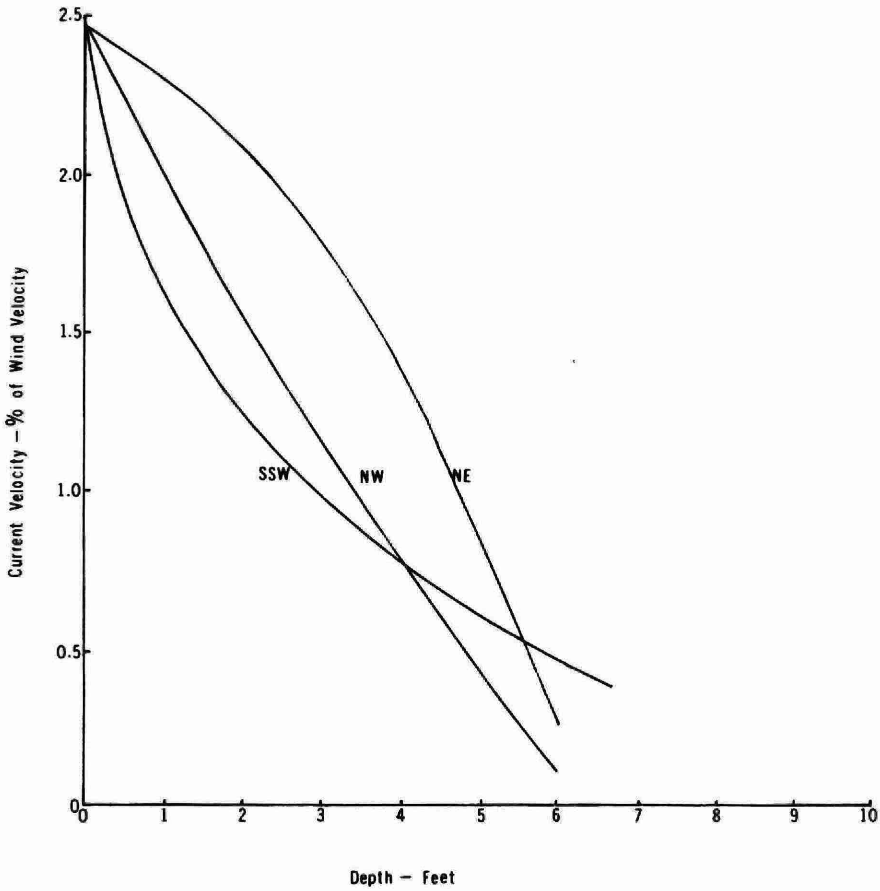

FIGURE 9 : Effect of depth on wind drift current velocity
Lakeview WPCP Outfall, July, 1966

2.5.0 DILUTION DETERMINATIONS

Since it was found that current speed and direction are largely dependent upon wind conditions, it may be assumed that under the influence of a wind in any direction other than between north and west, the currents will tend to carry the waste towards either the New Toronto or the Toronto Township water intake locations. Determinations of attainable dilutions may be determined for average and worst conditions using the meteorological data presented in the following Table 4.

a) Toronto Township Water Intakes

It was found that a basic current of 0.1 knots was flowing from an easterly direction in the Lakeview vicinity. During periods of calm, northeast to southeast winds, and light north winds, the wastewater mixture will be transported towards the Toronto Township water intakes. From Table 1 it may be seen that those conditions occur approximately 30% of the time. However, a change in wind direction is accompanied by a change in current direction with no greater lag period than 6 to 8 hours. Also initial effects of deep water currents influence inshore currents to some extent. Thus, one may assume that only 20 to 25% of the time will the wastewater mixture reach the Toronto Township intakes.

Maximum concentrations of waste will occur under a sustained wind of 38 mph in an east-northeast direction. Average conditions would occur at a wind speed of approximately 10 mph from an easterly direction.

Appendix B gives example computations of dilution determinations, taking into account both initial mixing and physical dilution.

Dilutions obtained under maximum and average conditions would be 99.4 and 99.8%, respectively.

TABLE IV

Toronto Island - 1965

Month	Time	39 mph & over	13-38 mph	1-12 mph	Calm	N	NE	E	SE	S	SW	W	NW
Jan.	13	0	19	12	0	1	2	7	0	1	8	8	4
	19	1	19	11	0	3	2	4	1	0	10	10	1
Feb.	13	0	18	8	1	0	2	1	3	6	5	5	4
	19	1	16	10	0	0	2	4	3	2	4	10	2
Mar.	13	0	16	15	0	1	7	3	3	3	2	5	7
	19	0	15	14	0	2	4	4	0	3	2	7	7
Apr.	13	0	14	16	0	1	5	5	0	8	4	3	4
	19	0	8	19	3	0	2	5	1	2	0	7	10
May	07	0	7	22	2	1	3	7	0	5	3	5	5
	13	0	10	20	1	1	2	8	2	10	3	4	0
	19	0	11	18	2	3	4	6	0	4	4	5	3
June	07	0	6	21	3	3	1	4	0	1	8	3	7
	13	0	13	17	0	1	0	9	2	8	4	2	4
	19	0	8	21	1	2	1	6	1	2	7	5	5
July	07	0	44	24	3	3	3	6	0	2	4	3	7
	13	0	8	23	0	0	0	5	5	11	4	4	2
	19	0	8	22	1	3	1	5	3	5	1	5	7
Aug.	07	0	3	23	5	1	3	4	1	2	6	4	5
	13	0	7	23	1	0	2	6	2	11	2	4	3
	19	0	8	20	3	1	0	3	3	4	5	5	7
Sept.	07	0	4	24	2	4	2	7	1	3	7	2	2
	13	0	11	19	0	0	4	11	2	9	1	2	2
	19	0	9	18	3	1	0	10	2	4	3	6	1
Oct.	07	0	9	18	3	1	5	1	1	2	9	4	4
	13	0	19	12	0	2	0	5	3	4	5	9	3
	19	0	10	19	2	0	3	4	0	3	5	9	5
Nov.	13	0	17	13	0	0	3	4	3	4	2	9	5
	19	0	8	20	2	2	1	9	0	1	5	6	4
Dec.	13	0	16	14	1	2	2	1	1	5	10	7	2
	19	0	11	15	2	0	2	2	0	4	6	9	3
Total		2	332	531	41	38	68	156	43	29	1 39	166	125
% of observations		0.22	36.5	58.5	4.5	4.4	7.9	18	5	15	16	19	14.5

b) New Toronto Water Intakes

Because of the basic current flowing from an easterly direction, under equal wind speeds, the wastewater mixture will move at a slower velocity towards the New Toronto intakes than towards the Toronto Township intakes. The distance of travel is also greater being 2.8 and 1.4 miles respectively. Dilutions will therefore be somewhat greater.

Using Table 4 as a guide, approximately 30% of the time the wastewater mixture will reach the New Toronto intakes.

Dilutions obtained under maximum and average conditions would be 99.5 and 99.9%, respectively.

2.6.0 CONCLUSIONS

The results of this study indicate that dilutions achieved by the outfall diffuser and lake turbulence are of such magnitude as to render the Lakeview WPCP effluent of negligible concentrations at both the Toronto Township and New Toronto water intakes.

The methods of determining these dilutions allows only an estimate of the actual dilutions, but are basically conservative.

Since we are dealing in this case, with a secondary treated effluent with chlorination provided, coliform concentrations, even after initial dilution only, are of such low concentrations that they are almost negligible compared to background coliform concentrations already present as shown in Figure 10.

The shoreline coliform concentrations have been calculated in Appendix C. These concentrations are very low compared to the background concentrations.

With the expansion of the plant from a design capacity of 5 MGD to 12 MGD, little change in dilutions will be effected because of the high diluting ratio of the lake water, provided efficient treatment along with chlorination is maintained.

2.7.0 RECOMMENDATION

Initial dilutions may be increased by one or both of two simple means. By decreasing the port size from 10" to 6" the initial dilution would be increased by close to 400% because of the higher exit velocities. By using only every second set of ports on the diffuser, the initial dilution could be increased by about 30% because of the decrease in interference between rising columns.

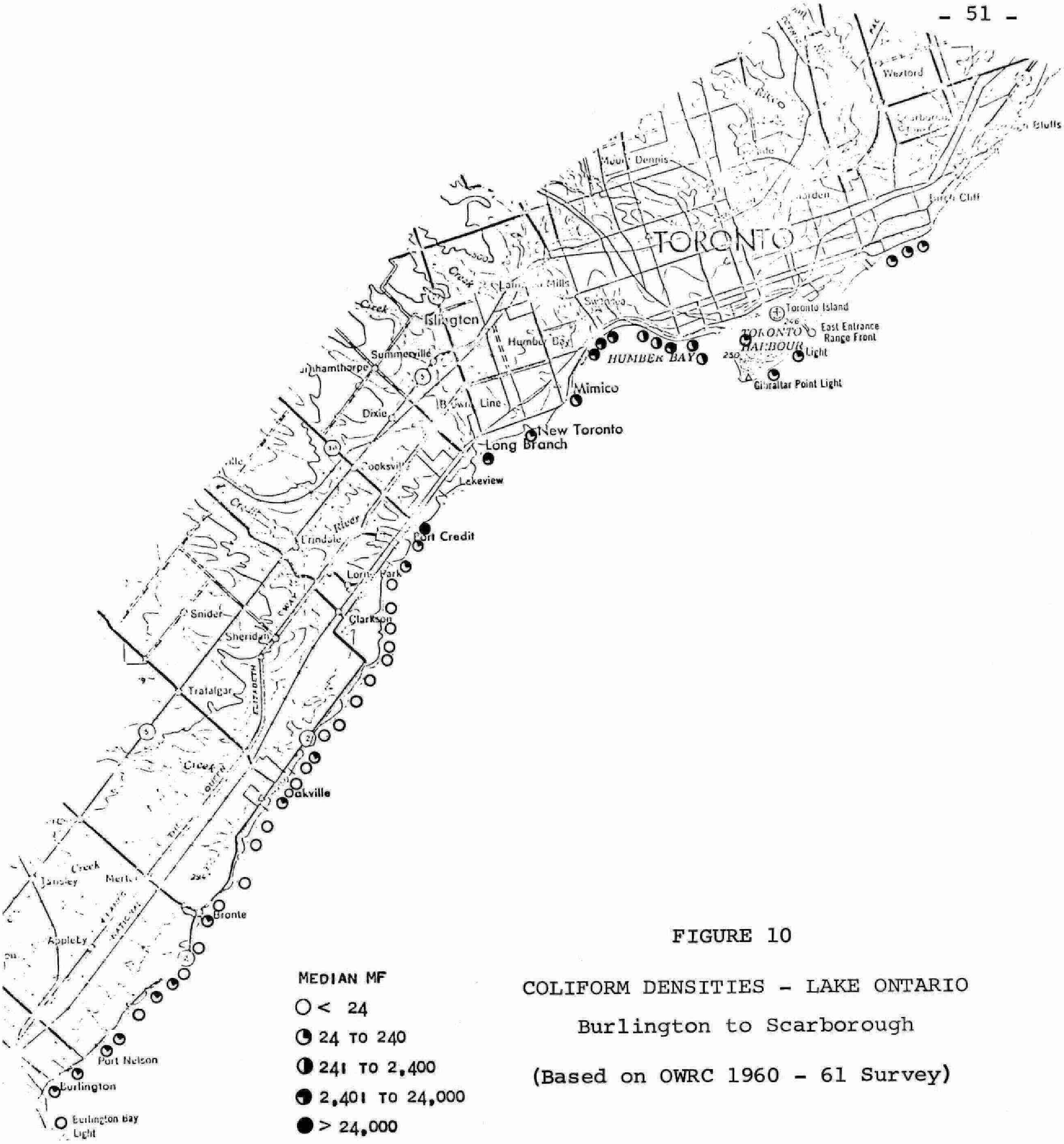

FIGURE 10

COLIFORM DENSITIES - LAKE ONTARIO

Burlington to Scarborough

(Based on OWRC 1960 - 61 Survey)

APPENDIX A

Drogue Movement Patterns

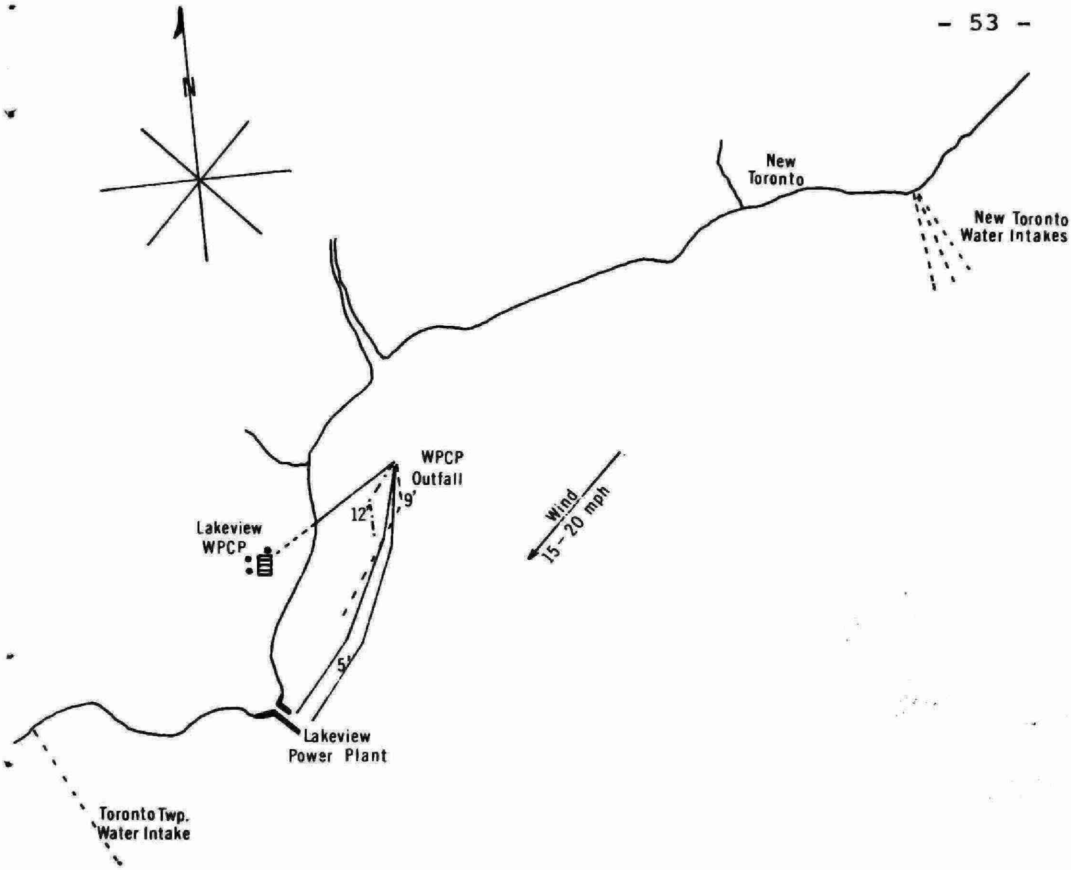

FIGURE 11

DIFFUSION STUDIES - Lakeview WPCP Outfall

July 19, 1966
Scale - 1" = 2,000'

Drogue Depth (ft) -	5	9	12
Speed (ft/min) -	30	17	5

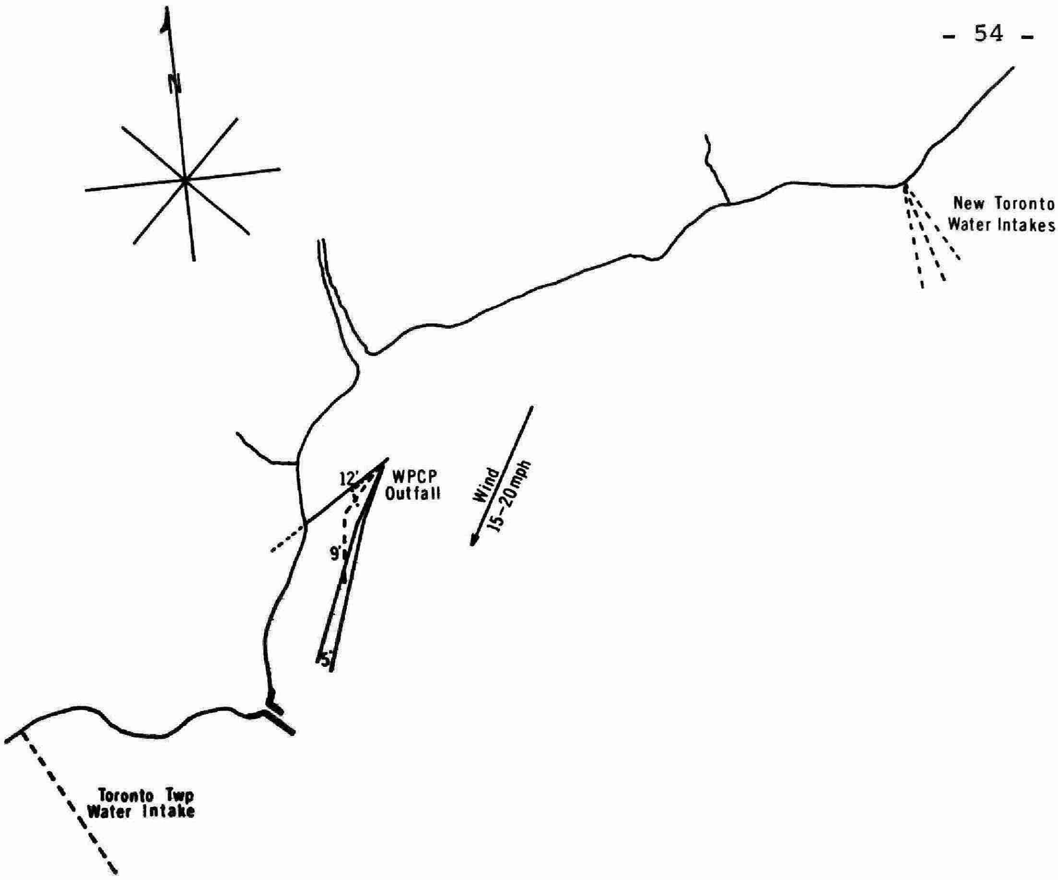

FIGURE 12

DIFFUSION STUDIES - Lakeview WPCP Outfall

July 20, 1967
Scale - 1" = 2,000'

Drogue Depth (ft) -	5	9	12
Speed (ft/min) -	22	10	2

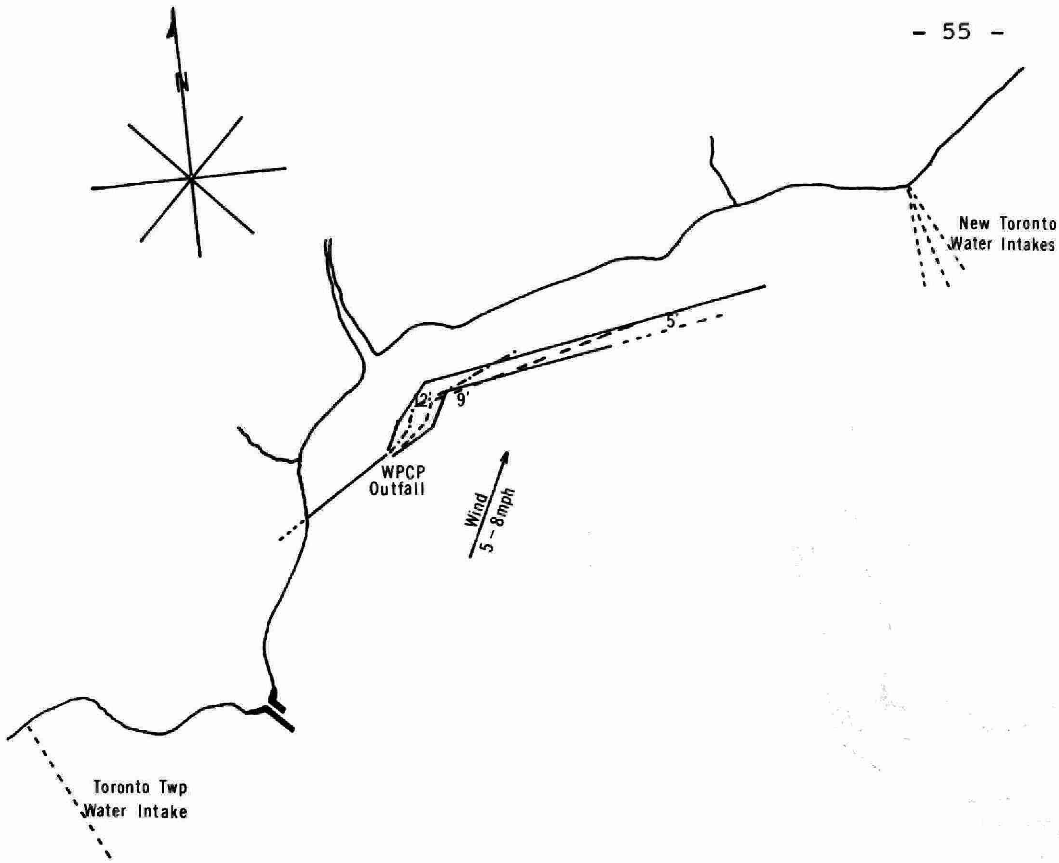

FIGURE 13

DIFFUSION STUDIES - Lakeview WPCP Outfall

July 22, 1966
Scale - 1" = 2,000'

Drogue Depth (ft) -	5	9	12
Speed (ft/min) -	6	4	2

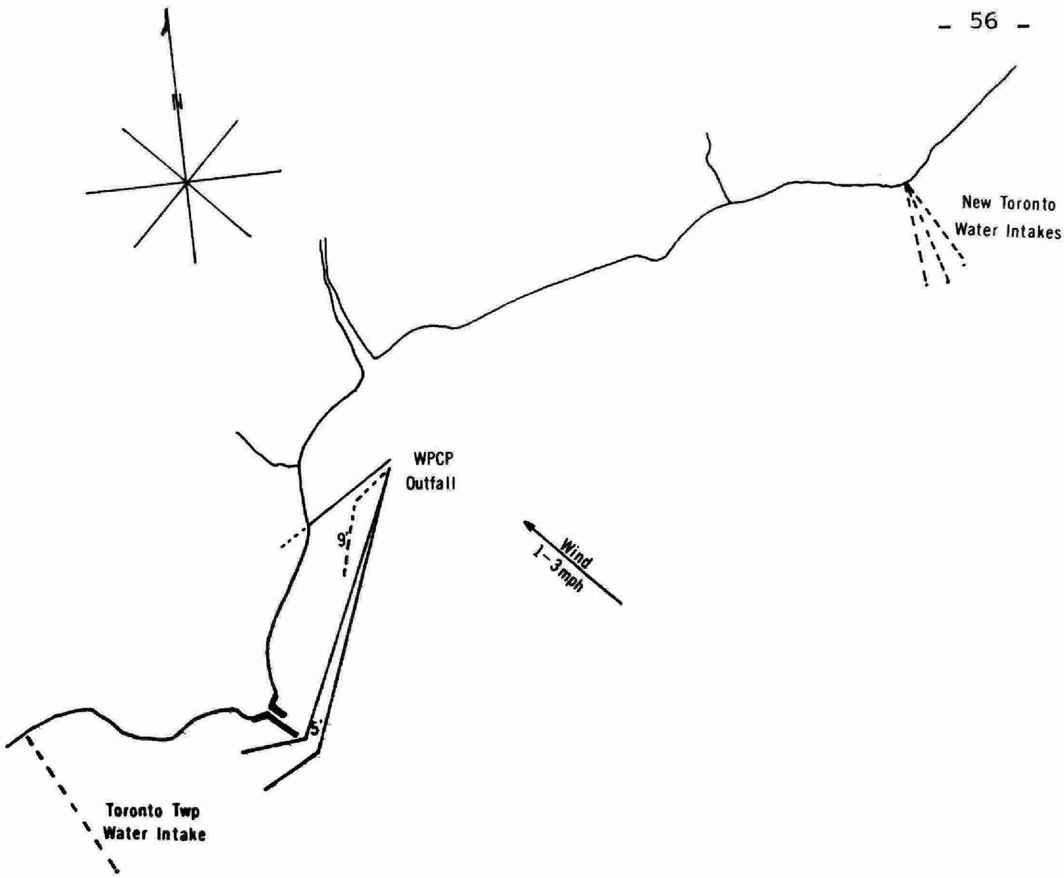

FIGURE 14

DIFFUSION STUDIES - Lakeview WPCP Outfall

August 9, 1966
Scale - 1" = 2,000'

Drogue Depth (ft) -	5	9	12
Speed (ft/min) -	17	6	-

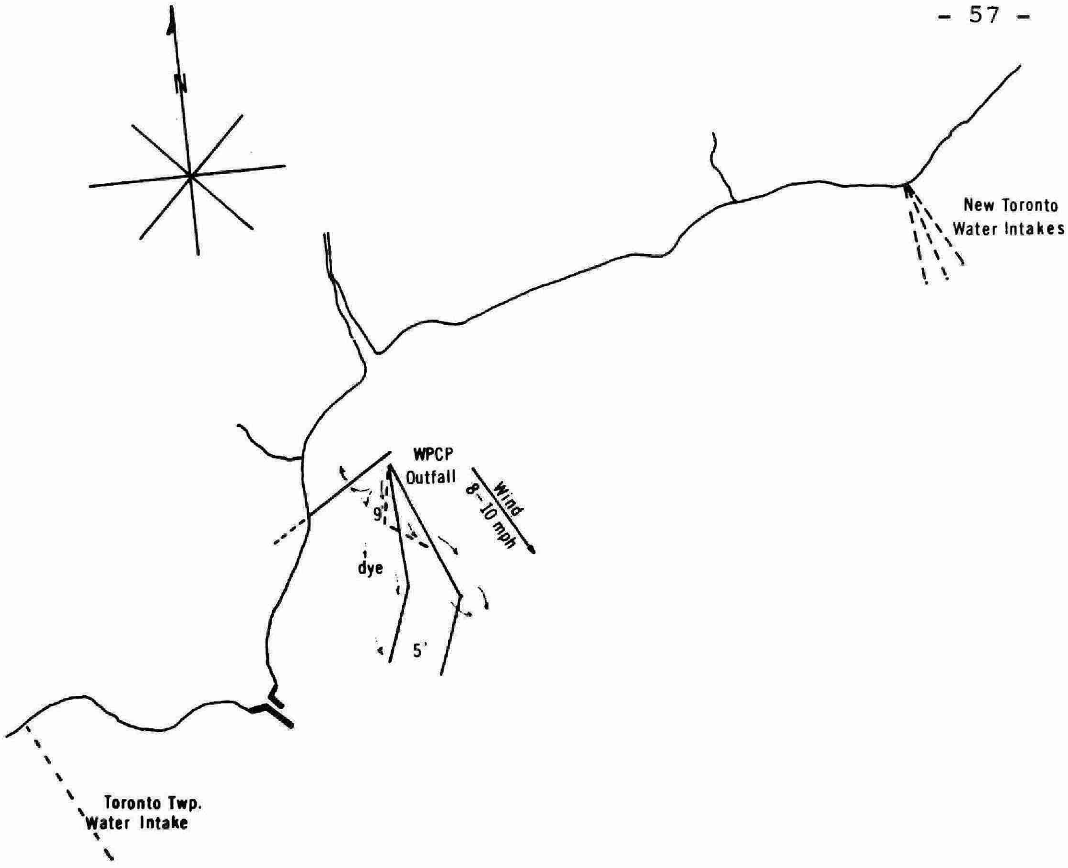

FIGURE 15

DIFFUSION STUDIES - Lakeview WPCP Outfall

August 17, 1966
Scale - 1" = 2,000'

Drogue and Dye Releases

APPENDIX B

Sample Computations of Water Intake Concentrations

1. Determination of Coliform concentrations due to Lakeview WPCP effluent at Toronto Township water intakes with conditions as outlined below:

Basic Conditions -

1.	Outfall diameter	4.5 ft
2.	Outfall length	2000 ft
3.	Effluent flow	7 mgd
4.	Temperature Lake	15^{o} C
	Waste	25^{o} C
5.	Depth of Diffuser	15 ft
6.	Straight diffuser with 9-10" ports	
7.	Length of Diffuser	64 ft
8.	Wind Velocity	NE at 38 mph
9.	Distance of Outfall	1.36 miles
10.	Coliform concentration of effluent	1000/100 ml

Initial Dilution

Density (s)

Lake (15^oC) = 0.9980

Waste (25^oC) = 0.997

Apparent Acceleration Due to Gravity (g')

$$g' = g\frac{\Delta S}{S} = 32.2\ \frac{(0.001)}{(0.997)} = 0.0323\ f5/sec^2$$

Depth - Diameter Ratio (Yo/D)

Yo = MWL - ½ outfall diameter

= 15 - 0.5 x 4.5 = 12.75 ft

Yo/D = 12.75/0.834 = 15.3

Froude Number (F)

Flow/port = 7 mgd/9 ports

= 7/9 x 1.85 = 1.44 cfs

Port area $= \pi(0.834)^2/4 = 0.545$ sq ft

Port Velocity (V) = 1.44/0.545 = 2.64 fps

$$F = \frac{V}{g'D} = \frac{2.64}{0.032 \times 0.834} = \frac{2.64}{0.163} = 16.2$$

Dilution (So) (Diffuser to Surface Only)

From Figure 3, for Yo/D = 15.3 and F = 16.2

So = 14

Jet Interference

Since the ports are located only 5'4" apart on the diffuser, there will be considerable interference between the rising plumes of waste effluent. Reductions in dilution due to this interference will be in the order of 30%.

Therefore So = 14 - (0.3 x 14) = 10

Initial dilution ratio = 1/10 = 0.1 = 10% (percent of effluent)

Physical Dilution

Wind produced current (Vw) - 1.5% of wind speed

Vw = 0.015 x 1.4667 = 0.835 fps

Current speed (U) - Wind produced + basic current

U = 0.835 + 0.1 x 1.689 = 1.0 fps

Initial field width (b)

$b = 64 \sin 25^{o} = 27$ ft

Turbulent Diffusion Coefficient (k)

Using Richardson's 4/3 law

$k = 0.01 \ (b)^{4/3} = 0.01 \ (27 \times 30.48)^{4/3} \ cm^2/sec$

$= 3.85 \times 0.01 \times 7,700 = 296 \ ft^2/hr$

$= 0.083 \ ft^2/sec$

Distance to intake (x)

x = 1.36 mi = 7,200 ft

Dilution (after initial mixing)

$$\frac{c}{Co} = \text{erf}\sqrt{\frac{3/2}{\left(1 + \frac{8kx}{Ub^2}\right)^3 - 1}}$$

$$= \text{erf}\sqrt{\frac{3/2}{\left(1 + \frac{8 \times 0.082 \times 7200}{1.0 \times 27^2}\right)^3 - 1}}$$

$$= 0.06$$

<u>Combined Dilution</u>

Intake Concentration = Initial dilution ratio x Physical dilution ratio

Effluent Concentration

= 0.1 x 0.06

= 0.006 = 0.60%

Intake Concentration

= 1000 x 0.006

= 6 coliforms/100 ml

Bacterial die off has not been included in this case because of the relatively short residence time.

2. Determination of coliform concentrations due to Lakeview WPCP effluent at New Toronto water intakes with conditions as outlined below.

Basic Conditions -

1.	Outfall diameter	4.5 ft
2.	Outfall length	2000 ft
3.	Effluent flow	7 MGD
4.	Temperature - Lake	15°C
	- Waste	25°C
5.	Depth of Diffuser	15 ft
6.	Length of Diffuser	64 ft
7.	Straight diffuser with 9 - 10" ports	
8.	Distance of outfall to intake	2.74 mi
9.	Wind velocity	WSW at 10 mph
10.	Coliform concentration of effluent	1000/100 ml

Initial Dilution

Density (s)

Lake (15°C) = 0.9980

Waste (25°C) = 0.997

Apparent acceleration due to gravity (g')

$$g' = g \frac{\Delta s}{s} = 32.2 \frac{(0.001)}{0.997} = 0.0323 \text{ ft/sec}^2$$

Depth - Diameter Ratio (Yo/D)

Yo = MWL - ½ outfall diameter

= 15 - 0.5 x 4.5 = 12.75 ft

Yo/D = 12.75/0.834 = 15.3

Froude Number (F)

Flow/port = 7 mgd/9 parts

= 7/9 x 1.85 = 1.44 cfs

Port area = $\pi\,(0.834)^2/_4$ = 0.545 sq ft

Port velocity (V) = 1.44/0.545 = 2.64 fps

$$F = \frac{V}{g'D} = \frac{2.64}{0.032 \times 0.834} = \frac{2.64}{0.163} = 16.2$$

Dilution (so) (Diffuser to surface only)

From Figure 3 for Yo/D = 15.3 and F = 16.2

So = 14

Physical Dilution

Wind produced current (Vw)

$$Vw = 0.015 \times 10 \times 1.4667 = 0.22 \text{ fps}$$

Current speed (U) - wind produced - basic

$$U = 0.22 - 0.1 \times 1.689 = 0.05 \text{ fps}$$

Initial field width (b)

$$b = 64 \cos 10^{o} = 11 \text{ ft}$$

Turbulent diffusion coefficient (k)

$$k = 0.01 \ (b)^{4/3} = 0.01 \ (11 \times 30.48)^{4/3} \text{ cm}^2/\text{sec}$$

$$= 3.85 \times 0.01 \times 2340 = 90 \text{ ft/hr}$$

$$= 0.025 \text{ ft}^2/\text{sec}$$

Distance to intake (x)

$$x = 2.74 \text{ mi} = 15{,}000 \text{ ft}$$

Dilution (after initial mixing)

$$\frac{c}{Co} = \text{erf} \sqrt{\frac{3/2}{\left(1 + \frac{8 \times 0.025 \times 15{,}000}{0.05 \times 11^2}\right)^3 - 1}}$$

$$= 1.11 \times 10^{-4}$$

Combined Dilution

$$\frac{\text{Intake Concentration}}{\text{Effluent Concentration}} = \begin{array}{l}\text{Initial dilution ratio x} \\ \text{Physical dilution ratio}\end{array}$$

$$= 0.1 \times 0.0001$$

$$= 1.0 \times 10^{-6}$$

Intake Coliform conc. = $1000 \times 1.0 \times 10^{-5}$

$= 1.0 \times 10^{-2}$ coliforms/100 ml

Bacterial Die-off - Residence time in this case will be approximately 80 hrs or 3 1/3 days. Thus die-off will again be negligible assuming T90 to be 12 days.

Appendix C

Shoreline coliform concentration due to the Lakeview WPCP outfall in Lake Ontario.

Basic Conditions -

1.	Outfall diameter	4.5 ft
2.	Outfall length	2000 ft
3.	Effluent flow	7 mgd
4.	Temperature Lake	15°C
	Waste	25°C
5.	Depth of diffuser	15 ft
6.	Length of diffuser	64 ft
7.	Straight diffuser with 9 - 10" ports	
8.	Wind velocity	S at 30 mph
9.	Coliform concentration of effluent	1000/100 ml

Initial Dilution

Same as calculated in Appendix B

So = 10

concentration ratio = 0.1

Physical Dilution

Wind produced current (Vw) - 1.5% of wind speed

$Vw = 0.015 \times 30 \times 1.4667 = 0.66$ fps

Current speed (U) - wind produced & basic current

$U = 0.66 + 0.10 = 0.76$ fps

Initial field width (b)

(b) = 64 ft

Turbulent diffusion coefficient (b)

$k = 0.01\ (64 \times 30.48)^{4/3}\ \mathrm{cm^2/sec}$

$= 962\ \mathrm{ft^2/hr}$

$= 0.26\ \mathrm{ft^2/sec}$

Distance to shoreline (x)

x = 2000 ft

Dilution (after initial mixing)

$$\frac{c}{Co} = \mathrm{erf}\sqrt{\frac{3/2}{\left(1 + \frac{8 \times 0.26 \times 2000}{0.76 \times 64^2}\right)^3 - 1}}$$

$= 0.354$

Combined Dilution

$$\frac{\text{Shoreline Concentration}}{\text{Effluent Concentration}} = \begin{array}{l}\text{Initial dilution ratio x} \\ \text{Physical dilution ratio}\end{array}$$

$$= 0.1 \times 0.35$$

$$= .035 = 3.5\%$$

Shoreline concentration = 1000 x .035

= 35 coliforms/100 ml